CHAPTER 3

SPILLWAY CREST

Section I. Introduction

3-l. General.

a. All spillways discussed in this manual require a spillway crest design. The crest and/or gates located near the crest provide the flow control through the spillway system. The capacity of the spillway is dependent upon the crest shape, crest length, and the hydraulic head. The hydraulic head is modified by approach conditions, pier and/or abutment effects, and submergence. The basic purpose of a spillway is to convey large floods through a project without incurring unacceptable damage either upstream or downstream from the spillway. The spillway design is accomplished in a manner that will minimize cost subject to providing:

(1) Sufficient crest length to convey the design discharge.

(2) Acceptable minimum pressures acting on the crest boundary.

(3) Acceptable maximum energy head on the spillway crest.

(4) Acceptable velocities and flow characteristics through the spillway system.

(5) Acceptable environmental and aesthetic conditions.

b. Engineering-economic investigations will usually show that a narrow spillway with high unit discharge is more economical than a wide spillway with moderate unit discharge. Thus, the most economic design will produce a spillway that includes a large energy head on the crest, a moderate design head, and a large unit discharge. Higher head spillways can create excessive abutment and pier contractions, cause energy dissipation problems, increase the possibilities of cavitation or pulsating nappe on the spillway crest, and create poor flow characteristics through the spillway system. The demand placed on the designer for economical designs requires the use of high head, high-efficiency spillways which, in turn, requires a sound design methodology. The objective of this chapter is to assist in providing this methodology.

Section II. Crest Characteristics

3-2. General. To provide a high-efficiency spillway and yet produce a safe, low-maintenance structure, the crest shape must provide a high discharge coefficient and fairly uniform and predictable pressures on the crest boundary. These constraints can best be met if the shape of the overflow spillway closely approximates that of a fully ventilated nappe of water flowing over a sharp-crested weir. The shape of the nappe is affected by the relative head on the weir, the approach depth and velocity, and the upstream slope of the weir. Experimental data gathered throughout a suitable range of these variables have led to the development of a spillway design methodology. The earliest attempts at fitting equations to lower nappe surfaces utilized the data of Bazin (item 6). Data developed by the US Bureau of Reclamation (USBR) (item 76) have served as a basis for most CE crest design procedures. Recent spillway investigations at WES (items 28, 32, and 33) have added considerably to the USBR data.

3-3. Crest Shape.

a. The complete shape of the lower nappe, which is also the spillway crest surface, is described by separating it into two quadrants upstream and downstream from the high point (apex) of the lower nappe. The apex is normally defined as the crest axis. The spillway crest shape is proportionally based on the design head H_d (see Chapter 2, Section II, for detailed definition of symbols used. The energy head H can be greater than, equal to, or less than H_d . The equation for the do&stream quadrant of the crest for all spillways can be expressed as

$$X^n = KH_d^{n-1}Y \qquad (3\text{-}1)$$

where

x = horizontal coordinate positive to the right, feet
n = variable, however usually set equal to 1.85
K = variable dependent upon P/H_d
Y - vertical coordinate positive downward, feet

Equation 3-l can be used to define the downstream crest shape for any P/H_d ratio by a systematic variation of K from 2.0 for a deep approach to 2.2 for a very shallow approach. See Plates 3-l and 3-2.

b. Difficulties existed when a single equation was fit to the upstream quadrant. The efficiency of the spillway is highly dependent on the curvature of the crest immediately upstream of the crest axis (items 32 and 51). A sudden change in curvature or a discontinuity not only disrupts the boundary layer but can also lead to flow separation and cavitation. Murphy (item 33) reported a three percent increase in the discharge coefficient when a small discontinuity between the upstream face and upstream quadrant was removed.

c. A general design procedure was advanced by Murphy (item 33) by which a sloping face spillway and a vertical face spillway can be designed in the same manner. For the upstream quadrant Murphy found that, by systematically varying the axes of an ellipse with depth of approach, he could closely approximate the lower nappe surfaces generated by USBR. Furthermore, any sloping upstream face could be used with little loss of accuracy if the slope became tangent to the ellipse calculated for a vertical upstream face.

d. The equation of the upstream elliptical shape is expressed as

$$\frac{X^2}{A^2} + \frac{(B - Y)^2}{B^2} = 1 \qquad (3\text{-}2)$$

where

X = horizontal coordinate origin at crest axis positive to the right
A = one-half horizontal axis of ellipse, feet
B = one-half vertical axis of ellipse, feet
Y = vertical coordinate origin at the crest axis positive downward

These three parameters (A, B, and K) then fully define the crest shape. Their variation with relative approach depth is given in Plate 3-2. This plate also includes a definition sketch.

3-4. Crest Discharge Coefficient. Discharge over a spillway crest is classified as either free flow or submerged flow. Free flow implies that the value of the discharge coefficient is not influenced by conditions downstream from the crest. Submerged flow occurs either when the tailwater is sufficiently high that a reduction in the discharge coefficient occurs, or when there is a change in the crest profile so close to the crest axis that the full benefits of the crest shape cannot be obtained. Flow over a spillway is governed by the relationship

$$Q = CL_eH_e^{1.5} \tag{3-3}$$

where Q is the rate of discharge and C is the discharge coefficient which is a measure of the efficiency of the spillway system. The discharge coefficient is a variable dependent upon generalized and site-specific factors. The factors which have been accounted for in generalized laboratory studies are the effect of relative approach depth P/H_d, the slope of the upstream face, the relative head on the crest H_e/H_d, crest submergence, and selected crest and abutment shapes. Site-specific factors such as flow angularity resulting from complex approach flow geometry or unusually shaped piers, for example, can be significant and must be investigated by a site-specific model study.

a. Free Discharge. Laboratory studies accomplished at WES (items 28 and 33) have defined spillway coefficients for free flow over a wide range of the following generalized factors: upstream slope, P/H_d, and H_e/H_d. Discharge coefficients reflecting these factors are given in Plates 3-3 and 3-4. Due to possible scale effects, discharges were not measured below $H_e/H_d = 0.4$. However, prototype experience has shown that spillway crests at very low heads exhibit the same discharge characteristics as a broad-crested weir. Therefore, for extrapolation purposes, the discharge coefficient should be equal to 3.08 as H_e/H_d approaches zero. As the P/H_d values decrease, and particularly for higher values of H_e/H_d, control of the flow begins to shift upstream, efficiency is lost, the discharge coefficient decreases, and the value of C again approaches that of a broad-crested weir (in this case a free overfall). Also to be noted is the characteristic increase in discharge coefficient for heads greater than design head. This is the concept of underdesigning the spillway crest to obtain greater efficiency. Underdesigning does not result in increased discharge coefficients with $P/H_d < 0.5$. The limitations of underdesigning the crest are dependent on the extent of negative pressure developed on the spillway crest. See Section IV, Crest Pressures, of this chapter.

b. Submerged Discharge. Submerged flow resulting from either excessive tailwater or changes in the crest profile will effectively reduce the free crest discharge coefficient. The reduction in the coefficient is dependent upon the degree of submergence. Due to the variance in the discharge coefficient, the effect of submergence cannot be described by a single relationship over the full range of the dependent variable. HDC 111-4 provides a discussion on tailwater submergency and provides a chart which defines a percent decrease in the unsubmerged crest coefficient for a full range of submergence. This chart is reproduced as Plate 3-5 for convenience. The curves shown on Plate 3-5 were based on three different test conditions: the approach and apron floors at the same constant elevation; both floors at the same elevation but varied with respect to the crest elevation; and the approach floor elevation held constant and the apron elevation varied. The percent decrease in the discharge coefficient was based on the unsubmerged discharge coefficient for each condition tested. EM 1110-2-1605 provides additional information on the effect of tailwater submergence on broad-crested spillways that are often used in conjunction with navigation dams. The reduction in the discharge coefficient resulting from crest geometry submergence is not as well defined as that for tailwater submergence. Abecasis (item 1) has accomplished some experiments that show when the chute tangent intersects the crest curve close to crest, a reduction in the discharge coefficient of two to eight percent will occur. The amount of reduction is dependent upon the location of the point of tangent intersection and the crest. When designs of this type are used and the discharge coefficient is critical, model studies will be necessary to verify the design.

Section III. Spillway Piers, Abutments, and Approach

3-5. General. Crest piers, abutments, and approach configurations of a variety of shapes and sizes have been used in conjunction with spillways. All of the variations in design were apparently used for good reasons. Not all of the designs have produced the intended results. Improper designs have led to cavitation damage, drastic reduction in the discharge capacity, unacceptable waves in the spillway chute, and harmonic surges in the spillway bays upstream from the gates. Maintaining the high efficiency of a spillway requires careful design of the spillway crest, the approach configuration, and the piers and abutments. For this reason, when design considerations require departure from established design data, model studies of the spillway system should be accomplished.

3-6. Contraction Coefficients. Crest piers and abutments effectively reduce the rate of discharge over the crest. The reduction in discharge is determined by the use of a contraction coefficient which, when applied in equation 2-2, defines the effective length of spillway crest. Conversely, additional crest length must be provided to offset the crest length reduction resulting from piers and abutments. Pier contraction coefficients have been determined from generalized model studies. Plate 3-6 shows plots of these contraction coefficients for five different pier nose shapes having the pier nose located in the same vertical plane as the spillway face and with $P/H_d > 1$. Plate 3-7 shows a plot of the contraction coefficient for a truncated elliptical pier nose that includes a bulkhead slot. This pier nose shape has been used on a number of the Columbia and Snake River projects. The contraction coefficients for the type 2 pier nose with piers extending upstream from the spillway face and $P/H > 1$ are shown in Plate 3-8. The contraction coefficients for $P/H_d < 1$ for the type 2 pier nose are shown in Plate 3-9. The contraction coefficients for the type 3 pier with an elliptical-shaped upstream crest with vertical or 1:1 upstream spillway slope and various P/H_d ratios are shown in Plate 3-10. The contraction coefficients for the variety of shapes and conditions show significant variation throughout a range of -0.075 to 0.10, thus the reason for careful consideration of the pier shape. Although some of these pier contraction coefficients show an increase in the efficiency of the spillway, it may be at the expense of lower pressures on the crest or undesirable flow conditions in the chute. As an example, the type 4 pier shown in Plate 3-6 provides increased efficiency throughout a wide range of H_e/H_d ; however, the flow conditions in the chute may be undesirable. Abutment contraction coefficients are not as available, as abutments are somewhat more site-specific. Plates 3-11 and 3-12 provide some basic information pertinent to abutments with adjacent concrete or embankment sections. (See paragraph 3-8 for additional information on abutment effects.)

3-7. Spillway Bay Surge. Surging of the water surface upstream from tainter gates has been observed during model studies of gated spillway crests on both high and low spillway crest. Model measurements indicate that water surface fluctuations as great as 10 feet with periods less than 10 seconds would occur in alternate bays of the prototype for certain combinations of gate bay width, w_b ; gate opening, G_o ; pier length, P_L defined as the distance from the upstream-most point of the gate face to the pier nose; and head on the crest, H_e . Model studies have shown that decreasing P_L , increasing W_b , or both, will effectively eliminate periodic surge. Excessive surging can be prevented by applying the following guidelines on spillway pier and gate bay design:

a. Low head spillways, $P/H_d < 1$

$$W_b \geqq 1.1H_c \quad \text{for} \quad P_L < 0.3W_b$$

or

$$W_b \geqq 1.25H_c \quad \text{for} \quad 0.3W_b < P_L < 0.4W_b$$

b. High head spillways, $P/H_d > 1$

$$W_b \geqq 0.8H_c \quad \text{for} \quad P_L < 0.3W_b$$

or

$$W_b \geqq 1.2H_c \quad \text{for} \quad 0.3 < P_L < 0.4W_b$$

where H_c is the maximum head on the crest where the gate controls the discharge. The maximum gate opening for which tainter gates will control the discharge should be taken as 0.625 times the head on the weir crest. By utilizing the spillway discharge curves for various gate openings, the maximum head on the weir crest for which the gates will control the discharge can be determined. These guidelines apply to all gated spillways regardless of the gate size. Due to the limited model tests used to develop the guidelines, model tests should be considered on those spillways which would operate with $G_o > 20$ feet and $H_e > 40$ feet. Conditions may dictate a design that is within the above limits, such as the increase in dam height which occurred at Chief Joseph Dam. At this project the model studies showed approximately five feet of surge alternating across the 19-bay spillway. Changing the dimensions of W_b or P_L was constrained by the existing structure so model studies were undertaken to evaluate surge suppressor designs. A simple design of two triangular concrete protrusions on the side of the pier upstream from the gate reduced the surge to well within acceptable limits without reducing the discharge characteristics of the spillway. See item 56 for detailed information.

3-8. Spillway Approach. Spillway approach configuration will influence the abutment contraction coefficient, the nappe profile, and possibly the flow characteristics throughout the spillway chute and stilling basin. There are three general configurations for the spillway approach, each of which requires a different treatment at the abutments in order to provide acceptable spillway characteristics.

a. Deep Approach. First, there is the high spillway where approach velocities are negligible. This condition usually exists at a spillway in the main river channel flanked by concrete nonoverflow sections. The P/H_d ratio for a deep approach spillway is defined as being greater than 1.0. The shape of the abutment adjoining concrete sections of a high head dam is a major factor influencing the abutment contraction coefficient. For this type of structure, the extension of the abutment upstream from the dam face to develop a larger abutment radius has provided improved flow characteristics in the end bays of the spillway (item 41). The abutment contraction coefficient curve shown in Plate 3-11 is applicable to this type of approach condition.

b. Shallow Approach. Second, there is the broad but relatively shallow approach that results in strong lateral currents at the abutments. This condition frequently is found at spillways in the river valley flanked by embankment sections. The P/H_d ratio for the shallow approach spillway is defined as being equal to or less than 1.0. When a spillway includes adjacent embankment sections, and particularly where approach velocities are appreciable, the configuration of the abutments and adjoining topography, the depth of approach flow, and the angularity of approach flow have significant influence on abutment contraction coefficients and flow characteristics. The embankment should not be carried at full height to the spillway training walls. Embankment wraparounds with concrete nonoverflow sections joining the top of the embankment to the spillway training walls should be considered. Abutment pier noses should not extend upstream of the face of the nonoverflow sections as this configuration has been noted to cause surging at the abutments. Rock dikes extending into the reservoir have been used to improve flow conditions at the abutments (item 66); however, optimum configurations are essential and can be developed only in a model study. An abutment contraction coefficient curve recommended for approach depths that are at least one-half of the design head and approach flow relatively perpendicular to the spillway are provided in Plate 3-12. Abutment contraction coefficients as large as 0.75 have been measured in model studies with very shallow approach ($P/H_d < 0.2$) and a curved approach channel (item 69).

c. Confined Approach. The third configuration results when the spillway is remote from the main dam and an excavated approach channel is required. In this type of approach, velocities may be high and flow distribution may be unequal but there will not be strong lateral currents at the abutments. When conditions require an excavated approach channel to the spillway, friction losses in the channel should be considered in determination of spillway capacity. Guidance for computing friction losses is given in Chapter 2. For confined channels the abutment contraction coefficient curve shown in Plate 3-11 may be used to account for abutment effects.

Section IV. Spillway Crest Pressures

3-9. General. Free discharge over a spillway crest designed to the shape discussed in paragraph 3-3 will develop pressures on the concrete boundary somewhat inversely proportional to the H_e/H_d ratio. When H_e/H_d is nearly one, the pressures on the crest are essentially atmospheric. As H_e/H_d increases, crest pressures drop below atmospheric. These negative pressures are the reason for the increase in the discharge coefficient over that of a ventilated sharp-crested weir. A reasonable approximation of the crest pressures will provide the data necessary for structural stability analysis for certain design cases. Crest pressure calculations will also provide the hydraulic design guidance on the limiting pressures that crest underdesigning yields prior to reaching pressures where cavitation damage occurs. Previous recommendations (item 77) have stated that H_e/H_d should not exceed 1.33 when underdesigning a spillway crest. Bauer and Beck (item 5) and Abecasis (item 2) have shown that the actual minimum pressure fluctuation level in relation to local atmospheric pressure is what leads to cavitation. Vacuum tank observations by Abecasis (item 1) indicated that cavitation on the crest would be incipient at an average pressure of about -25 feet. Fluctuations and duration of actual pressures at or near absolute, not the average pressure on the crest boundary, are the cause of cavitation damage. A spillway crest should be designed so that the maximum expected head will result in average pressures on the crest no lower than -15 feet of water at sea level and 40-degree Fahrenheit temperature. The -15 feet of water must be adjusted to account for elevation and water temperature at the spillway crest site. HDC 000-2 and 001-2 will provide data to assist in this adjustment. For spillways with and without piers, Plates 3-13 and 3-14, respectively, show a relationship between H_e and H_d defining the maximum limit of underdesign allowed based on the recommended minimum crest pressure of -15 feet of water. The curves for -25 feet and -20 feet of water are also shown on these plates for comparison.

3-10. Controlled and Uncontrolled Crests. A controlled crest is one that includes gates which are used to control the flow; the uncontrolled crest is one unencumbered by gates. Pressures on controlled and uncontrolled crests with vertical 1:l upstream sloped faces with P/H_d values of 0.25, 0.5, and 1.0 were investigated at WES (item 28). At $P/H_d = 0.25$, pressures were measured for $H_e/H_d = 0.5$ and 1.0 only. Use of an underdesigned crest with a

P/H_d value as low as 0.25 does not result in a significant increase in the discharge coefficient above $H_e/H_d = 1.0$. WES investigations included two piers placed on a model crest. The pier nose used for all crests was the type 3 shown on Plate 3-6. The pier nose was located in the same plane as the upstream face for the vertical spillway. For the 1V:1H upstream slope, the pier nose location was determined by maintaining the same distance from pier nose to crest axis as used in the vertical upstream faced crest. See item 28 for detailed information on crest pressure distribution for various P/H_d ratio spillways, with and without a sloping upstream face, and various H_e/H ratios. For spillways that include piers, the minimum pressure along the pier limits the amount of underdesigning permissible. When a crest with piers is designed for negative pressures, the piers must be extended downstream beyond the negative pressure zone in order to prevent aeration of the nappe, nappe separation or undulation, and loss of the underdesign efficiency advantage. For preliminary design purposes, the approximate range of the dimensionless horizontal distance from the crest axis (X/H_d) where pressures were found to return to positive, are as follows:

H_e/H_d	X/H_d
1.17	0.1-0.4
1.33	0.7-0.9
1.5	1.1-1.5

Section V. Upper Nappe Profile

3-11. General. The upper nappe profile or the water surface profile for free flow over a spillway crest with or without piers is of acute interest in the design of sidewalls adjacent to the spillway crest, equipment bridges over the spillway crest, and spillway gate trunnion location. The nappe profile unencumbered by crest piers is somewhat different from one with piers. The upper nappe profile will also be modified by the direction of the approach flow with respect to the crest axis. Procedures to determine nappe profiles have been derived from experimental work based on specific conditions involving P/H_d and H_e/H_d ratios, spillways with and without piers, and approach flow perpendicular to the crest axis. These procedures provide a sound basis for design of nappe profile-related features. When hydraulic conditions vary somewhat from the experimental conditions, or the upper nappe profile is critical to the design, model studies to accurately determine the profile are recommended.

3-12. Nappe Profile. The design procedure used to determine the upper nappe profile is based on generalized experimental data. Upper nappe profile data for two spillway conditions are presented. The first is for high spillways with negligible approach velocities as discussed in paragraph 3-8a. The second condition is for low spillways with appreciable approach velocities as discussed in paragraphs 3-8b and c.

a. High Spillways. Plate 3-15 shows generalized data in the form of dimensionless coordinates of the upper nappe profile in terms of the design head for H_e/H_d ratios of 0.50, 1.00 and 1.33 without the influence of crest piers. Plates 3-16 and 3-17 show the dimensionless coordinates for the same conditions with the influence of crest piers.

b. Low Spillways. Plates 3-18 through 3-20 show generalized data in the form of dimensionless coordinates of the upper nappe profile along the center line and along the edge of a crest pier in terms of the design head. These data are presented for H_e/H_d ratios of 0.05, 1.0, and 1.5 for P/H_d ratios of 1.0, 0.50, and 0.25.